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NATIONAL DAM SAFETY PROGRAM, CITY PARK LAKE DAM (MO 30588), UPP-ETC(U)
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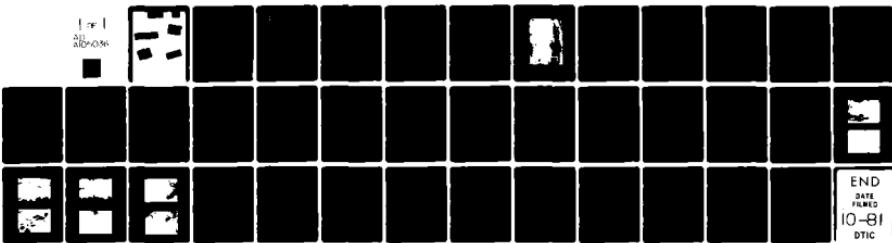
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.		



DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 NORTH 12TH STREET
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IN REPLY REFER TO

SUBJECT: City Park Lake Dam (Mo. 30588)

This report presents the results of field inspection and evaluation of City Park Lake Dam (Mo. 30588).

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe because of heavy tree growth on the crest of the dam and on both the upstream and downstream embankment slopes, seepage on the downstream embankment slope and at the toe of the dam, and a seriously inadequate spillway that will pass only 3 percent of the probable maximum flood.

SUBMITTED BY:

SIGNED

Chief, Engineering Division

21 MAR 1979

Date

APPROVED BY:

Colonel, CE, District Engineer

22 MAR 1979

Date

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CITY PARK LAKE DAM
CRAWFORD COUNTY, MISSOURI

MISSOURI INVENTORY NO. 30588

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY

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St. Louis, Missouri
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St. Louis, Missouri

PREPARED FOR

ST. LOUIS DISTRICT, CORPS OF ENGINEERS
NOVEMBER, 1978

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam	City Park Lake
State Located	Missouri
County Located	Crawford County
Stream	Tributary To Stater Creek
Date of Inspection	August 15, 1978

City Park Lake Dam, No. 30588 was inspected using the "Recommended Guidelines for Safety Inspection of Dams". These guidelines were developed by the Chief of Engineers, U. S. Army, Washington, D.C., with the help of Federal and state agencies, professional engineering organizations, and private engineers. The resulting guidelines are considered to represent a consensus of the engineering profession.

City Park Dam was visually inspected by an interdisciplinary team of engineers from Kenneth Balk & Associates, Inc. and Shannon & Wilson, Inc. The purpose of the inspection was to make a preliminary assessment of the general condition of the dam with respect to safety in order to determine if, in the opinion of the interdisciplinary team, the dam poses recognizable hazards to human life or property. This assessment is based solely upon data made available and visual evidence observed during the site visit.

To make a complete assessment of the safety of the dam would require detailed studies and engineering analyses beyond the scope of this preliminary assessment.

Based on the criteria in the guidelines, the dam is in the high hazard potential classification, which means that loss of life and appreciable property loss could occur in the event of failure of the dam. The estimated damage zone extends three miles downstream of the dam. Within the damage zone are three to four houses and two county road crossings. City Park Dam is in the intermediate size classification since it is greater than 40 feet high but less than 100 feet high.

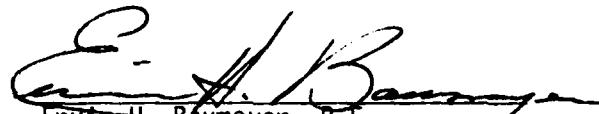
The inspection and evaluation indicate that the spillway of City Park Lake does not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. City Park Lake is an intermediate size dam with a high hazard potential, required by the guidelines to pass the PMF. Considering the high hazard potential to loss of life and property downstream of the dam, the outlet facilities of City Park Lake should be able to pass the PMF without overtopping the dam. However, it was determined that the spillway will only pass approximately 3 percent of the PMF without overtopping the dam.

Since the outlet facilities for City Park Lake are not capable of passing the PMF without overtopping the dam, and causing failure, the spillway is considered inadequate.

The evaluation of City Park Lake also indicated that the spillway will not pass the 100-year flood; that is, a flood having a 1 percent chance of being equalled or exceeded during any given year.

Serious deficiencies visually observed by the inspection team were sloughing, erosion, seepage, a very thick stand of trees on the downstream slope, large trees on the crest and the upstream slope of the embankment and a large slide on the right abutment. In addition, the downstream embankment slope is steeper than that generally used for dams of this height. Other deficiencies found were the lack of seepage records, operational records, seepage and stability analyses comparable to the requirements of the recommended guidelines, and seismic stability analyses.

We recommend that prompt action be taken to correct the serious deficiencies described. A detailed report discussing each of these deficiencies is attached.



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Overview of Lake and Dam

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
CITY PARK LAKE DAM - ID NO. 30588

TABLE OF CONTENTS

<u>Paragraph No.</u>	<u>Title</u>	<u>Page No.</u>
SECTION 1 - PROJECT INFORMATION		
1.1	General	1
1.2	Description of Project	1-2
1.3	Pertinent Data	2-3
SECTION 2 - ENGINEERING DATA		
2.1	Design	4
2.2	Construction	4
2.3	Operation	4
2.4	Evaluation	4
SECTION 3 - VISUAL INSPECTION		
3.1	Findings	5-6
3.2	Evaluation	7
SECTION 4 - OPERATIONAL PROCEDURES		
4.1	Procedures	8
4.2	Maintenance of Dam	8
4.3	Maintenance of Operating Facilities	8
4.4	Description of Any Warning System in Effect	8
4.5	Evaluation	8
SECTION 5 - HYDRAULIC/HYDROLOGIC		
5.1	Evaluation of Features	9
SECTION 6 - STRUCTURAL STABILITY		
6.1	Evaluation of Structural Stability	10
SECTION 7 - ASSESSMENT/REMEDIAL MEASURES		
7.1	Dam Assessment	11
7.2	Remedial Measures	11-12

APPENDIX

A Hydrologic and Hydraulic Analysis Methodology

LIST OF PLATES

<u>Plate No.</u>	<u>Title</u>
1	Vicinity Topography
2	Location Map
3	Top of Dam Elevations
4	Top Of Dam Profile And Section

LIST OF PHOTOGRAPHS

<u>Photo No.</u>	<u>Title</u>
1	Overview of Lake and Dam
2	Crest of Dam
3	Upstream Face and Crest of Dam
4	Upstream Embankment Rip Rap Protection
5	Spillway Entrance Showing Erosion On Left Shoulder
6	View of Spillway Bottom
7	Slide Area in Right Abutment
8	Downstream Face of Embankment Shot From Toe

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the St. Louis District, Corps of Engineers, District Engineer directed that a safety inspection of the City Park Lake Dam be made.

b. Purpose of Inspection. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon data made available and visual inspection, in order to determine if the dam poses hazards to human life or property.

c. Evaluation Criteria. Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief of Engineers, in "Recommended Guidelines for Safety Inspection of Dams". These guidelines were developed with the help of several Federal agencies and many State agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances.

(1) The dam is an earth structure built in a small narrow valley in the northeastern part of Crawford County near Sullivan, Missouri. Topography adjacent to the valley is rolling to steep. Soils in the vicinity of the dam consist mainly of silty clays with pieces of chert, dolomite and sandstone. Bed rock is exposed in places upstream and downstream of dam. Topography in the vicinity of dam is shown on Plate 1.

(2) A spillway, three feet deep and eleven feet wide, cut partially in bedrock and partially in soil, is located on the left abutment. The spillway is lined with mortared cut stone.

(3) Pertinent physical data is given in paragraph 1.3.

b. Location. The dam is located in the northern portion of Crawford County, Missouri, as shown on Plate 2. The lake formed by the dam is on the Missouri-Crawford County Sullivan quadrangle sheet in the NW 1/4 of Section 20, T40N, R2W.

c. Size Classification. Criteria for determining the size classification of dams and impoundments are presented in the guidelines referenced in paragraph 1.1c above. Based on these criteria, this dam and impoundment is in the intermediate size category.

d. Hazard Classification. Guidelines for determining hazard classification are presented in the same guidelines as referenced in paragraph c. Based on referenced guidelines, the Corps of Engineers has determined that this dam is in the High Hazard Classification and thus has been selected by the Corps of Engineers for a Phase I inspection.

e. Ownership. The lake and dam are owned by the City of Sullivan, Missouri.

f. Purpose of Dam. The dam forms a recreational lake.

g. Design and Construction History. The inspection team was unable to find at the City of Sullivan any design data on this dam. It is reported to have been a WPA Project, completed in 1941. This information was obtained from Messrs. John Waller and Earl Schultz, who were employed on the project, both residents of Sullivan, Missouri.

h. Normal Operating Procedure. No operating records were found. Outflow passes through an uncontrolled spillway. Normal rainfall, runoff, transpiration, evaporation and spillway discharges all combine to maintain a relatively stable water surface elevation.

1.3 PERTINENT DATA

a. Drainage Area - 53 acres.

b. Discharge at Damsite.

(1) Spillway 19.2 cfs. at maximum pool.

(2) Estimated experienced maximum flood - approximately at top of dam.

c. Elevation (U.S.G.S.)

(1) Top of dam - 952.7 (Min.).

(2) Invert of spillway - 952.4.

(3) Spillway crest - 952.4.

(4) Streambed at centerline of dam - 900 \pm .

(5) Maximum tailwater - unknown.

d. Reservoir. Length of maximum pool - 1200 feet \pm .

e. Storage (Acre-feet).

(1) Normal - 111.8

(2) Maximum - 114.7

f. Reservoir Surface (Acres).

- (1) Top of dam - 6.2.
- (2) Spillway crest - 5.7.

g. Dam.

- (1) Type - earth embankment.
- (2) Length - 250 feet.
- (3) Height - 53 feet maximum.
- (4) Top width - 16 feet.

(5) Side Slopes - (Measured by slope meter/inclinometer in degrees and converted to ratios).

- (a) Downstream - 1.75 H to 1 V.
- (b) Upstream - 3 H. to 1 V. to waterline.

(6) Zoning - unknown

(7) Impervious core - unknown

(8) Cutoff - unknown

(9) Grout curtain - unknown

h. Diversion and Regulating Tunnel. None.

i. Spillway.

- (1) Type - Grouted rock rectangular channel with fish screen.
- (2) Bottom width - 10.5 feet.
- (3) Side slopes approximately 6 H. to 1 V.
- (4) Average longitudinal slope - +7%.
- (5) Crest elevation - 952.4.

j. Emergency Overflow Spillway - None.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

No design data were found to be readily available.

2.2 CONSTRUCTION

Based on information gathered from local residents, the dam was completed in 1941 by the WPA.

2.3 OPERATION

No records of the maximum loading on the dam were available.

2.4 EVALUATION

a. Availability. No engineering or geological data were readily available.

b. Adequacy. No engineering data were made available to make a detailed assessment of the design, construction, and operation. The lack of seepage and stability analyses comparable to the requirements of the Recommended Guidelines is considered a deficiency which should be corrected. An engineer experienced in the design of dams should be retained to perform detailed seepage and stability analyses.

c. Validity. No valid engineering data on design or construction were available.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

A. General. A visual inspection of City Park Dam was made on August 15, 1978. Personnel making inspection were employees of Kenneth Balk and Associates, Inc. and Shannon and Wilson, Inc. of St. Louis, Missouri and included civil, geotechnical, structural engineers and an engineering geologist. Specific observations are discussed below.

B. Dam. The inspection team observed the following items at the dam.

The dam is extremely overgrown with trees, brush and other vegetation. Medium and large size trees are present on the upstream slope and the crest of the embankment. The alignment of the crest is slightly bent and the crest also has a low spot (approximately 18 inches lower than the average crest elevation). No cracking was observed, however, the dense vegetation may be concealing any cracks.

The downstream slope is densely covered with small and medium size trees, brush and vines and is wet and spongy throughout. The dense vegetation may be concealing such deficiencies as animal burrows, sloughing, and slope instability. A zone of seepage is present across the face of the slope, about 35 to 40 feet from crest, as measured along the slope. Total seepage from this zone is about five to ten gallons per minute. Seepage, estimated to be 5 to 10 gallons per minute, was also observed on both the left and right abutments near the toe of the embankment. The area downstream of the toe is marshy and wet.

The dam has very little freeboard (approximately 0.6 feet) at the low spot mentioned above. The upstream slope is apparently faced with fitted slabs of rock which vary in size and where visible were found to be in good condition.

C. Appurtenant Structures. A spillway, which is cut part in soil and a part in sandstone bedrock, is located at the juncture of the left abutment and the dam embankment. The spillway is lined with fitted mortared rock slabs which vary in size. This uncontrolled spillway is the only structure existing to control pool level. The lake level at the time of our visit was just below the spillway sill thus no water was flowing through the spillway.

The spillway outlet channel, which is three feet deep, eleven feet wide and 175 foot long, is lined with fitted and mortared rock slabs, varying in size. A section of the lining (one foot x eight feet) on the left wall near the base and approximately fifty to sixty feet downstream of spillway bridge, has deteriorated. The downstream end of the spillway outlet is also breaking up and eroding. A footbridge is placed across the outlet channel, and a fish screen is located in the channel downstream from the bridge. There is a thick growth of grass and brush growing in the spillway channel.

D. Reservoir Area. Sloughing and/or erosion, approximately 30 feet wide and 19 feet long, was observed on the left bank of the reservoir approximately 25 feet from the spillway. No other wave wash, excessive erosion or slides were observed along the shores of reservoir.

E. Damsite Geology.

1) Left Abutment: Outcrops of coarse to medium grained, moderately hard buff colored sandstone are present on left abutment, downstream of the embankment. This sandstone is moderately weathered, moderately massive, moderate to sparsely jointed. Small cavities ranging in size from 1/2 to 1 inch in diameter are present. The inner surface of cavities are coated with yellow limonitic material.

2) Right Abutment: Outcrops of bedrock are visible on the right abutment. These outcrops consist of mainly medium to fine grained, buff to light-brown, moderately fractured, moderately weathered, medium hard to hard sandstone. The sandstone is composed of fine to medium grained quartz sand which characteristically is subrounded and frosted. The fresh surface of this sandstone is light-yellow to tan in color. The sandstone is interbedded with cherty dolomite. The upper-most bed is two feet three inches thick, massive sandstone, buff colored with a few fractures, moderately weathered, moderately hard. Underlying is a highly fractured zone three feet seven inches thick, consisting of alternate beds of sandstone and cherty dolomite. At places lenses of pure white quartz sand are present. Below this bed is a massive sandstone bed, having similar properties as that of the upper most sandstone bed.

The following two types of joints are observed in this formation:

Dip:	Vertical, strike 15 degrees NW
Dip:	85 SW, Striked 70 degrees NE
Width of Joints:	1/8 inch to 1/4 inch
Joint Spacing:	Moderate
Filling Material:	Yellow limonitic clayey material

A slide, approximately 50' by 40' in plan, was observed on the right abutment approximately 50 feet below the crest of the abutment and downstream of the dam (see photo no. 7). The slide seems to be some years old judging by the small trees growing in the slide area, however, based on a recently overturned tree may still be active. It is our opinion, based on the shape of the slide, that it is a rotational movement of an earth mass.

3.2 EVALUATION

The conditions observed are serious enough, in our opinion, to indicate a need for prompt remedial action. The extremely dense vegetative cover made a thorough visual inspection impractical and other deficiencies may be present, but not visible.

Although no slope instability was detected, the downstream slope is steep, wet and soft throughout and its long term stability cannot be assured. The large slide on the right abutment is a deficiency which may affect the stability of the dam and should be evaluated.

In the opinion of the inspection team, the services of a professional engineer experienced in the design of the dams should be obtained to evaluate the overall stability of the dam and the abutments.

The existing seepage and marshy condition on the downstream face of the embankment, if left uncorrected, may adversely affect the stability of the dam.

All of the above deficiencies should be corrected; however, vegetative growth is of such size and extent that removal should be attempted only under the direction of and with observation by, an engineer experienced in the design and construction of dams. Indiscriminate clearing could create an unsafe condition, possibly leading to failure of the dam.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

There are no controlled outlet works for the dam, therefore, no regulating procedures exist. The pool is controlled by rainfall, run off, evaporation and the capacity of the uncontrolled spillway.

4.2 MAINTENANCE OF DAM

No maintenance records were made available, however, the number and size of trees and other vegetation on the crest and slopes of the dam suggest that no regular maintenance has been or is being performed. The slide mentioned in paragraph 3.1 is unrepaired.

4.3 MAINTENANCE OF OPERATING FACILITIES

No operating facilities exist.

4.4 DESCRIPTION OF ANY WARNING SYSTEMS

The inspection team is not aware of any existing warning system for this dam.

4.5 EVALUATION

It is our opinion that in view of the serious deficiencies observed, a warning system should be established immediately and maintained. The large trees and other vegetation on the dam are deficiencies that should be corrected, however, this should only be done under the direction of an experienced engineer to avoid creating an unsafe condition.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. There were no hydraulic and hydrological design data made available.

b. Experience Data. The drainage area and lake surface area are developed from USGS Sullivan Mo. Quadrangle, dated 1969. The spillway and dam layout are from surveys made during the inspection.

c. Visual Observations.

(1) The spillway and exit channel are located at the left or east abutment and are in need of repair; i.e., mortared rock lining deterioration, vegetation, etc. Spillway discharges, in our opinion, would not endanger the integrity of the dam.

d. Overtopping Potential. The spillway has been found to be inadequate to pass the Probable Maximum Flood (PMF) without overtopping the dam. The probable maximum flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region.

For the PMF, the dam would be overtopped to a maximum height of approximately 2.5 feet with a duration of overtopping of approximately 16.8 hours and a maximum discharge rate of 753 cfs.

For 50% of the PMF, the dam would be overtopped to a maximum height of approximately 1.8 feet with a duration of overtopping of approximately 15.4 hours and a maximum discharge rate of 344 cfs.

In our opinion, failure of the dam may be expected to occur as a result of overtopping for these lengths of time.

The spillway has been found to be adequate to pass a flood of approximately three percent (3%) of the PMF.

The spillway has also been found to be inadequate to pass the 100-year flood, which has a 1% chance of being equalled or exceeded at least once during any given year.

The estimated damage zone extends three miles downstream of the dam. Within the damage zone are three to four houses and two county road crossings.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observed conditions which can affect the structural stability of this dam have been discussed in Section 3.

b. Design and Construction Data. No design or construction data relating to the structural stability of the dam were available.

c. Operating Records. No appurtenant structures requiring operation exist at the dam.

d. Post-Construction Changes. No post-construction changes were observed.

e. Seismic Stability. The City Park Dam is located in Seismic Zone 1 near the boundary of Zone 2. The seismic stability of this dam could not be evaluated due to lack of engineering design data; however, to our knowledge, an earthquake of the magnitude that may reasonably be expected in Seismic Zone 1 has not caused a structural collapse of a dam of this size.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. The conditions observed are serious enough, in our opinion, to indicate a need for prompt remedial action. In the opinion of the inspection team, the services of a professional engineer experienced in the design of dams should be obtained to further evaluate and develop remedial actions for the serious deficiencies noted, i.e., trees on the embankment, brush in the spillway, deteriorated condition of the spillway lining, the inadequate capacity of the spillway, seepage and the stability of the embankment and the abutments.

b. Adequacy of Information. Due to the lack of engineering design and construction data, the conclusions of this report are based on performance and visual conditions. The lack of seepage and stability analysis comparable to the requirements of the Recommended Guidelines is a deficiency which should be corrected.

c. Urgency. The deficiencies noted are serious enough to require prompt action.

7.2 REMEDIAL PROCEDURES

a. O&M Procedures. The following O&M procedures are recommended:

(1) The spillway should be cleared of vegetation and the fish screen repaired.

(2) The spillway outlet channel should be repaired.

(3) The seepage should be monitored to determine the quantity of flow and sedimentation, if any, and corrective action should be designed by an experienced engineer.

(4) Remove trees, brush and vines on the embankment. However, growth is of such size and magnitude that this and all following actions should be done only under the direction of an engineer experienced in the design and construction of dams. Indiscriminate clearing could create an unsafe condition.

(5) Spillway capacity and/or height of dam should be increased to pass 100 percent (100%) of the Probable Maximum Flood.

(6) A detailed investigation into the cause of the slide on the right abutment should be initiated and appropriate remedial measures designed and implemented.

(7) Stability and seepage analyses should be performed by a professional engineer experienced in the design and construction of dams.

(8) Up-to-date records of all future maintenance and repairs should be kept.

(9) The dam should be periodically inspected by an engineer experienced in the design and construction of dams, and records kept of these inspections.

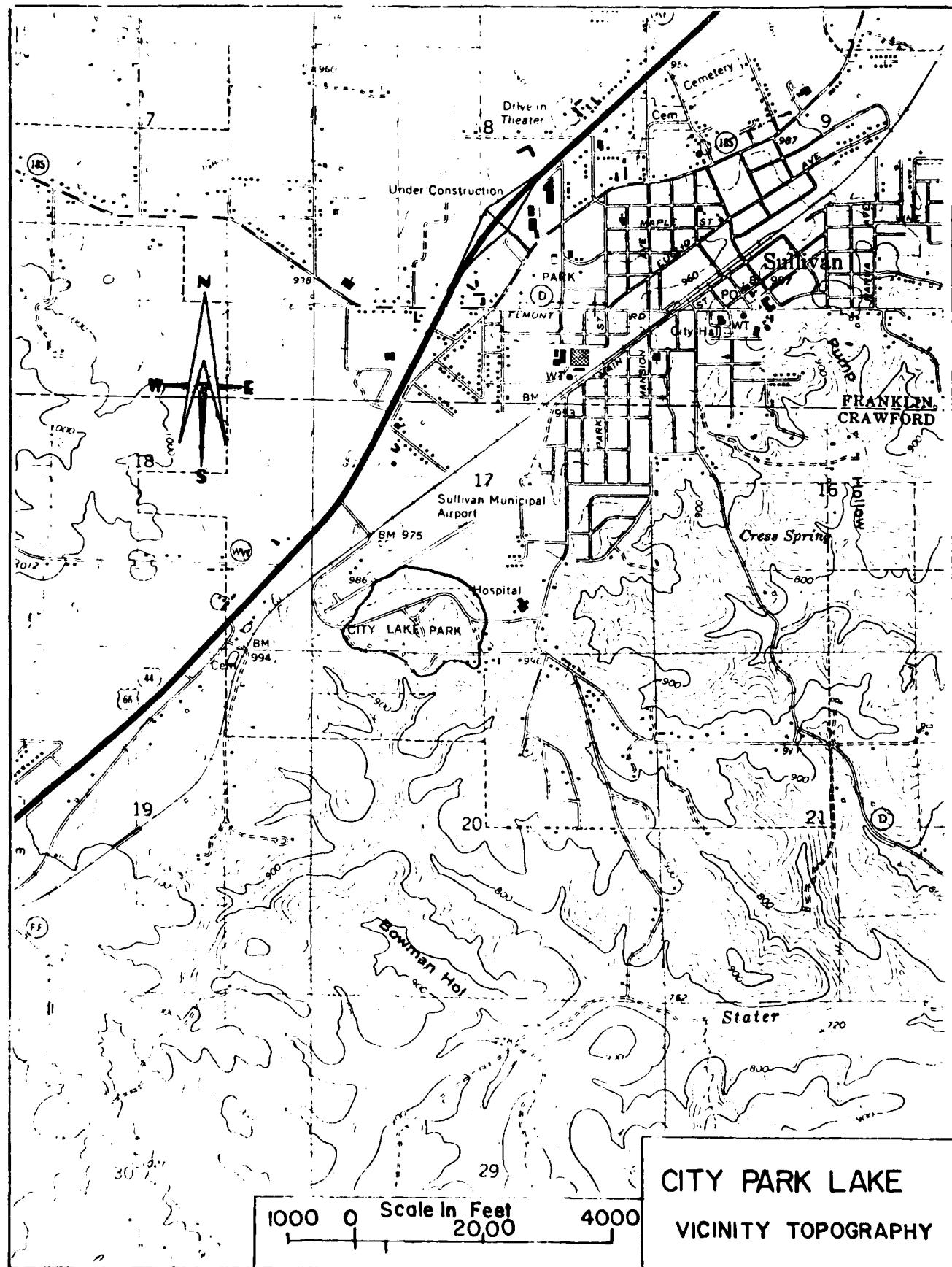
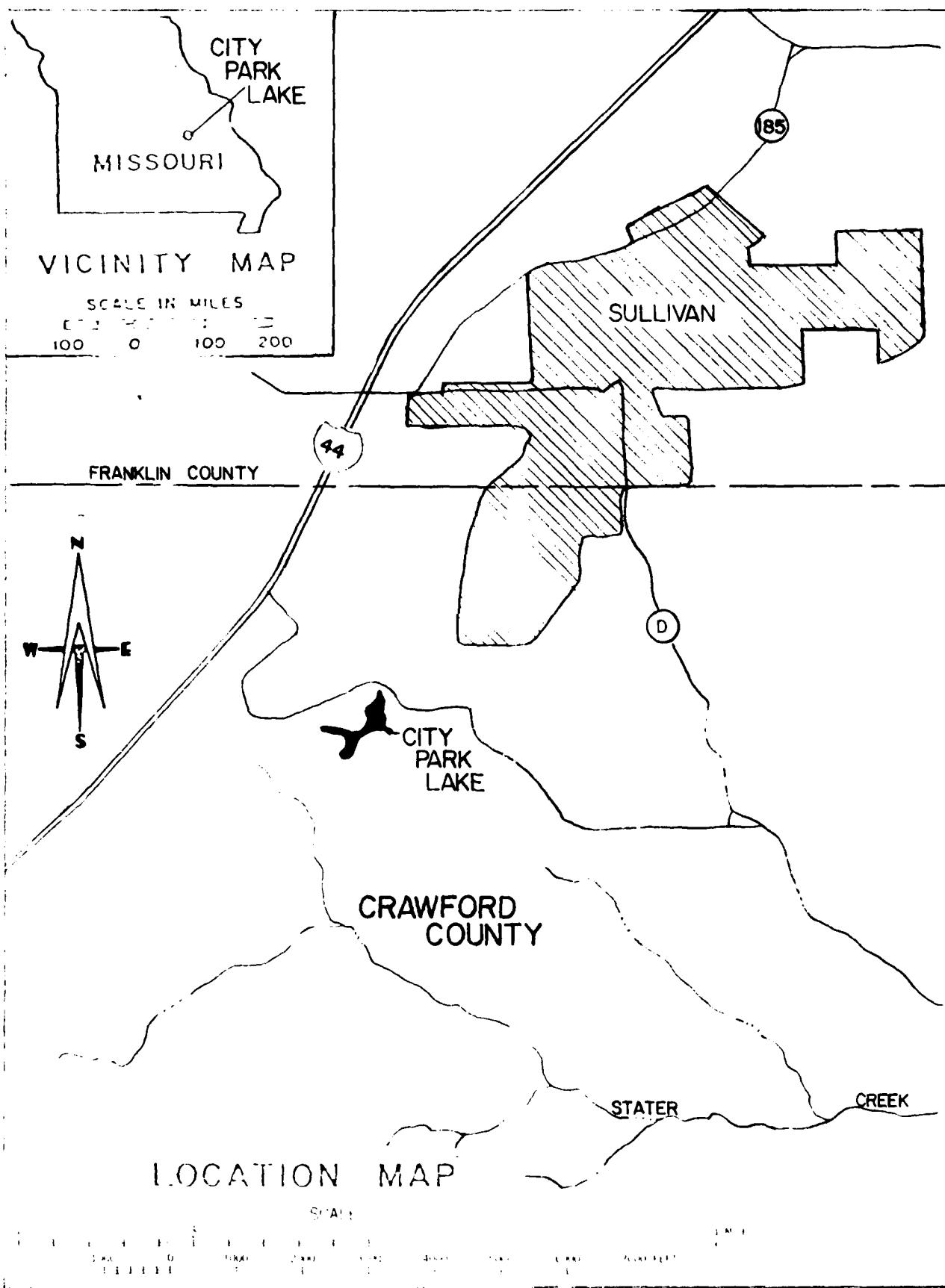
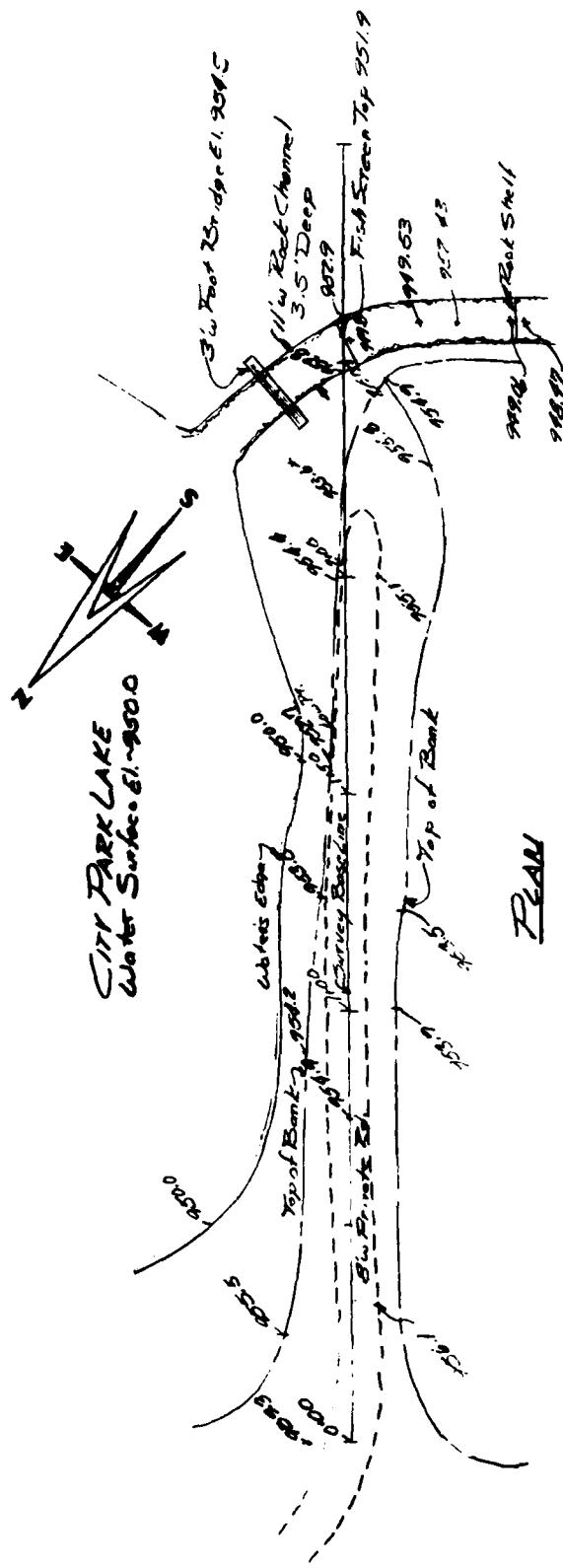


PLATE I

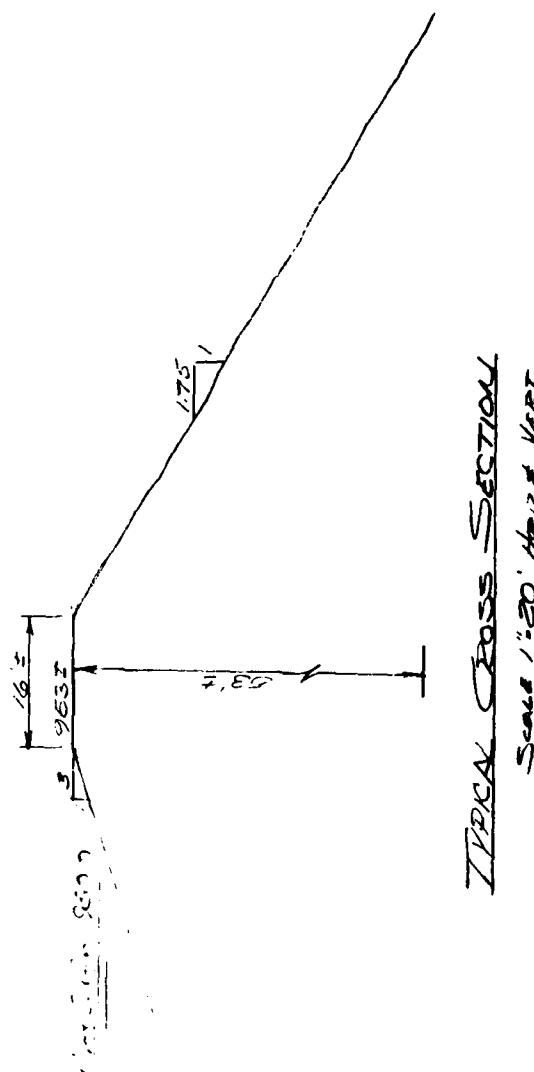
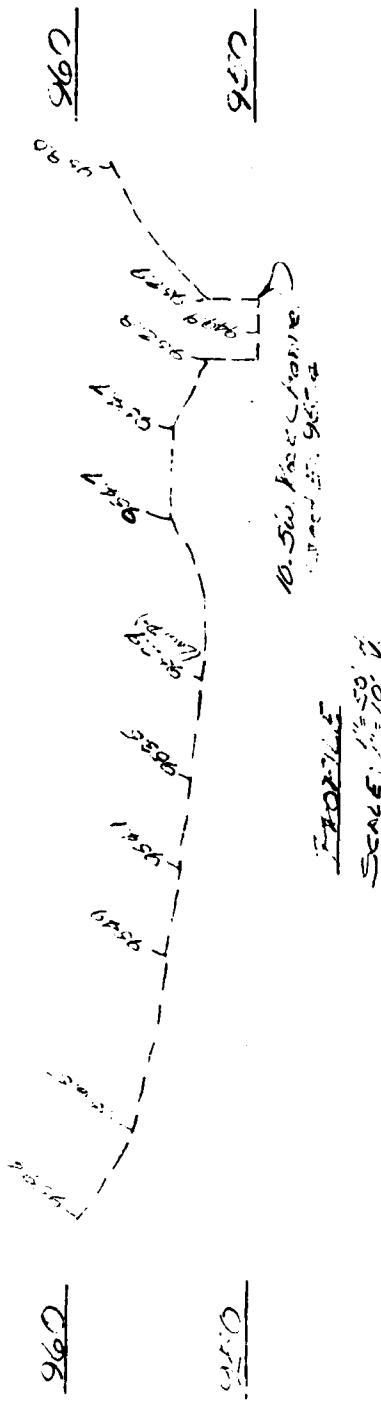




CITY PARK LAKE

TOP OF DAM ELEVATIONS

SCALE: 1": 40' PLATE 3



CITY PARK LAKE DAM PROFILE and CROSS SECTION

PLATE 4



PHOTO 1 Overview of Lake and Dam

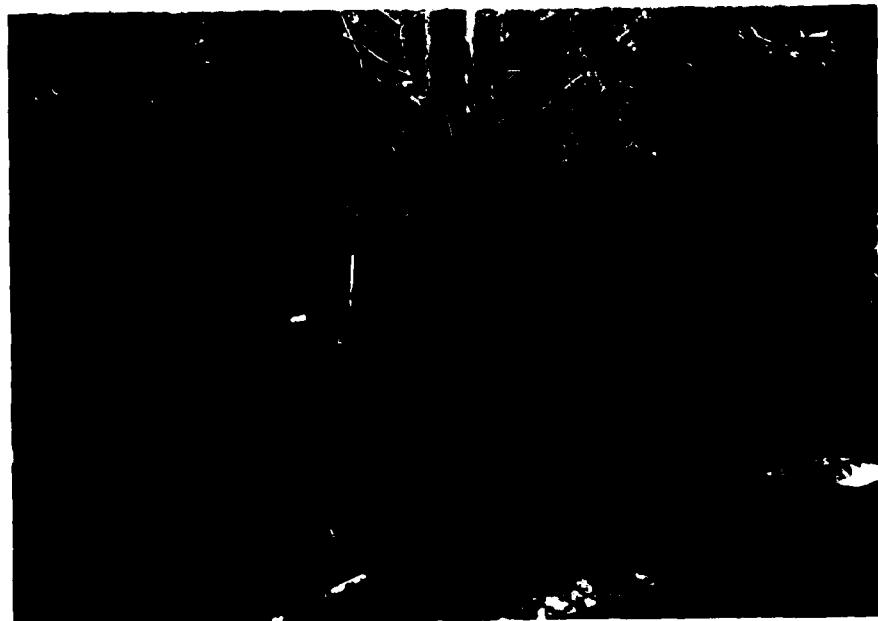


PHOTO 2 Crest of Dam



PHOTO 3 Upstream Face and Crest of Dam



PHOTO 4 Upstream Embankment Rip Rap Protection



PHOTO 5 Spillway Entrance showing Erosion on Left Shoulder

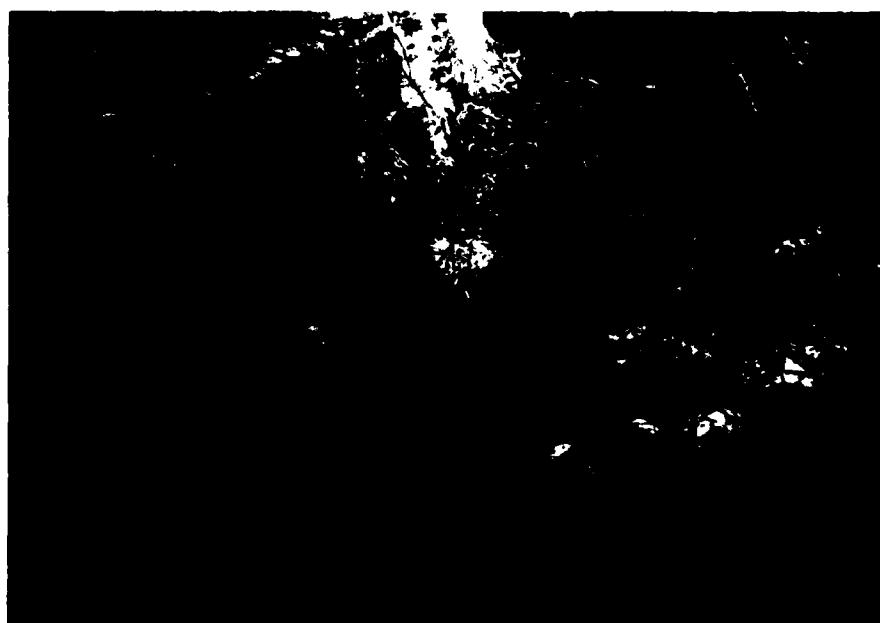


PHOTO 6 View of Spillway Bottom

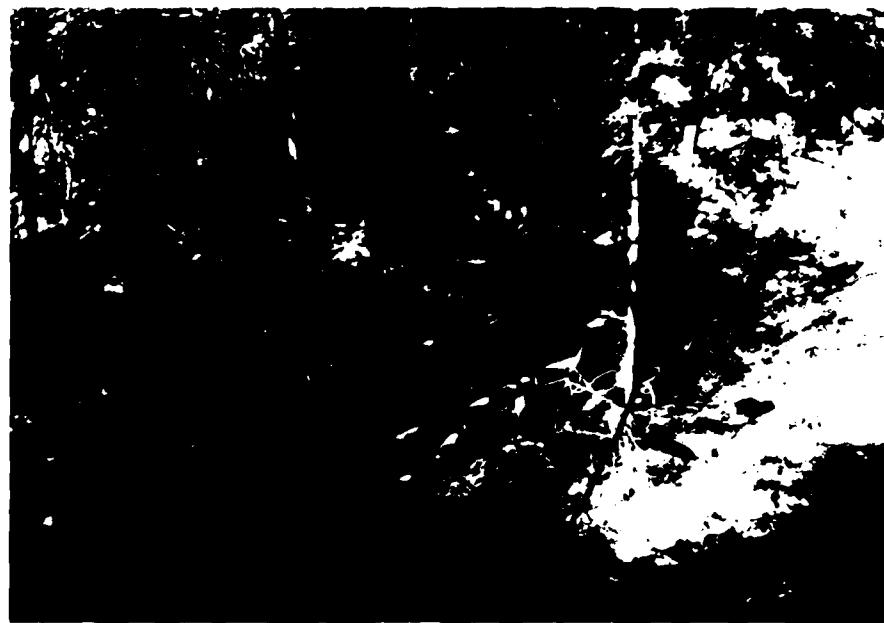


PHOTO 7 Slide Area in Right Abutment

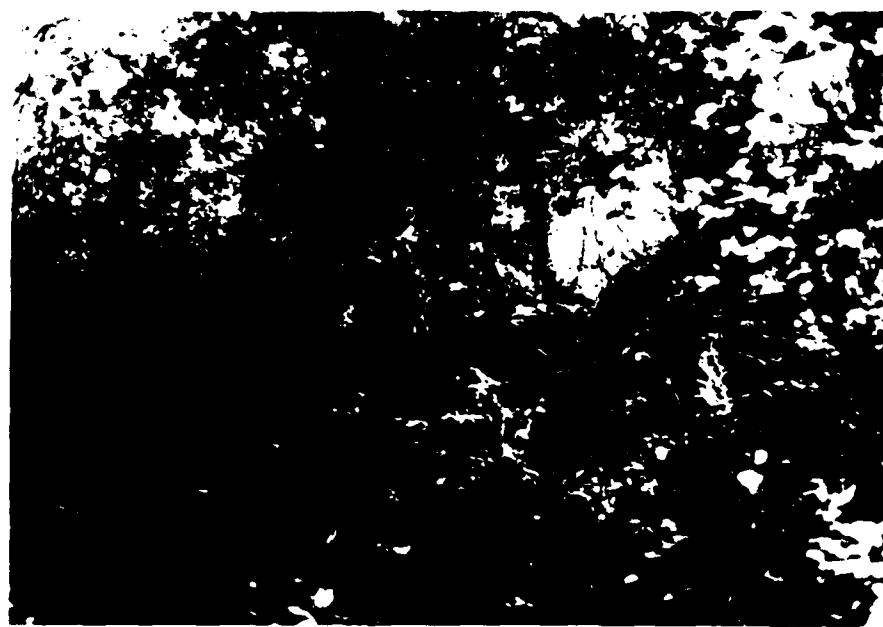


PHOTO 8 Downstream Face of Embankment shot
from Toe

APPENDIX A
HYDROLOGIC AND HYDRAULIC ANALYSIS METHODOLOGY

HYDROLOGIC AND HYDRAULIC ANALYSIS METHODOLOGY

1. The hydrologic analysis used in development of the overtopping potential is based on applying a hypothetical storm to a unit hydrograph to obtain the inflow hydrograph for a reservoir routing. The Probable Maximum Precipitation is derived and determined from regional charts prepared by the National Weather Service in "Hydrometeorological Report No. 33." Reduction factors have not been applied. A 24-hour storm duration is assumed with the total rainfall depth distributed over 6-hour periods in accordance with procedures outlined in EM 1110-2-1411 (SPF Determination). The maximum 6-hour rainfall period is then distributed to hourly increments by the same criteria. Within-the-hour distribution is based upon NOAA Technical Memorandum NWS HYDRO-35. The nonpeak 6-hour rainfall periods are distributed uniformly. All distributed values are arranged in a critical sequence by the SPF criteria. The final inflow hydrograph is produced by utilizing the Soil Conservation Service dimensionless unit hydrograph using Hydrologic Soils Group "C", Antecedent Moisture Condition III, and SCS CN #88 used to determine rainfall excess.

Lag time was estimated using methods outlined in "Design of Small Dams", by the United States Department of The Interior, Bureau of Reclamation. Using this source, lag time is taken as 60% of the time of concentration.

Time of concentration was estimated utilizing methods outlined in the source quoted above, supplemented by data obtained during field investigation. The results of the field investigation indicated that a minimum time of 20 minutes should prevail over a lesser value obtained using the methods outlined in the quoted source. For this lake, a lag time of 0.20 hours was therefore selected.

2. The reservoir routing is accomplished by using Modified Puls routing techniques wherein the flood hydrograph is routed through lake storage. Hydraulic capacities of the outlet works, spillway, and crest of dam are used as outlet controls in the routing. Storage in the pool area is defined by an elevation-storage capacity curve. The hydraulic capacity of the outlet works, spillway, and top of dam are defined by elevation-discharge curves.

3. Dam overtopping analysis has been conducted by hydrologic methods for this dam and lake. This computation determines the percentage of the PMF hydrograph that the reservoir can contain without the dam being overtopped. An output summary in the hydrologic appendix displays this information as well as other characteristics of the simulated dam overtopping.

4. The above methodology has been accomplished for this report using the systemized computer program HEC-1 (Dam Safety Version), July 1978, prepared by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California. The numeric parameters estimated for this site are listed in the attached computer printout. Definitions of these variables are contained in the "User's Manual" for the computer program.

5. The inflow hydrograph was routed through the reservoir using HEC-1's Modified Puls option. Releases were calculated for: 1) the principal spillway, and, 2) the flow over the top of the dam. These releases were then combined at each of their respective elevations.

Flow through the rectangular spillway and outlet channel was calculated by writing the Bernoulli equation between the lake water surface and the energy gradient elevation in the channel.

With flow in the channel at normal depth, and using the lake water surface as the datum, and, assuming velocity of approach to be zero, the following equation is written:

$$H = E.G. + h_e$$

Where H = difference between lake water surface and the Energy Gradient Elevation in the channel,

E.G. = Energy Gradient Elevation in the channel = depth of flow + $\frac{V^2}{2g}$

$$h_e = \text{Entrance loss} = k_e \frac{V^2}{2g}$$

$$\text{Where } k_e = 0.5$$

The equation can then be simplified as follows:

$$\text{Stage} = I.E. + d_f + 1.5 \frac{V^2}{2g}$$

Where I.E. = Invert Elevation

d_f = normal flow depth for a given discharge

Stage = Pool elevation

Flow over the top of dam was calculated using the weir flow equation:

$$Q = CL(H)^{1.5}$$

where: C = Varies with head as outlined in "Handbook of Hydraulics" by Horace Williams King, revised by Ernest F. Brater.

L = Length in feet (varies with water surface)

H = Head of water in feet (varies with water surface)

Q = Discharge in cfs

1.01	11.20	136	.07	.00	.46	1.01	23.20	.280	.02	.02	.00
1.01	11.25	137	.07	.00	.45	1.01	23.25	.281	.02	.02	.00
1.01	11.30	138	.07	.00	.45	1.01	23.30	.282	.02	.02	.00
1.01	11.35	139	.07	.00	.45	1.01	23.35	.283	.02	.02	.00
1.01	11.40	140	.07	.00	.45	1.01	23.40	.284	.02	.02	.00
1.01	11.45	141	.07	.00	.45	1.01	23.45	.285	.02	.02	.00
1.01	11.50	142	.07	.00	.45	1.01	23.50	.286	.02	.02	.00
1.01	11.55	143	.07	.00	.45	1.01	23.55	.287	.02	.02	.00
1.01	12.00	144	.07	.00	.45	1.02	0.00	.288	.02	.02	.00
							SUM	33.80	32.30	1.50	210600
								(859.)	(820.)	(38.)	(596.35)

HYDROGRAPH AT STANFLOW FOR PLAN 1. RT10 1

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	944.	228.	73.	73.	21051.
CMS	27.	6.	2.	2.	596.
INCHES					32.77
MM					832.33
AC-FT	649.37	832.33	832.33	832.33	832.33
THOUS CU M					145.
AC-FT	113.	145.	145.	145.	145.
THOUS CU M					179.
					179.
					5.

HYDROGRAPH AT STANFLOW FOR PLAN 1. RT10 2

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	28.	7.	2.	2.	632.
CMS	1.	0.	0.	0.	18.
INCHES					30.
MM					98.
AC-FT	19.48	24.97	24.97	24.97	24.97
THOUS CU M		3.	4.	4.	4.
AC-FT		4.	5.	5.	5.
THOUS CU M		7.	9.	9.	9.

HYDROGRAPH AT STANFLOW FOR PLAN 1. RT10 3

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	47.	11.	4.	4.	1053.
CMS	1.	0.	0.	0.	30.
INCHES					1.64
MM					1.64
AC-FT	32.47	41.62	41.62	41.62	41.62
THOUS CU M		6.	7.	7.	7.
AC-FT		7.	9.	9.	9.
THOUS CU M		7.	9.	9.	9.

HYDROGRAPH AT STANFLOW FOR PLAN 1. RT10 4

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	76.	18.	6.	6.	1684.
CMS	2.	1.	0.	0.	48.
INCHES					2.62
MM					2.62
AC-FT	51.95	66.59	66.59	66.59	66.59
THOUS CU M		9.	12.	12.	12.
AC-FT		11.	14.	14.	14.
THOUS CU M		11.	14.	14.	14.

UNITED COMPUTING SYSTEMS, INC.

SUMMARY OF DAM SAFETY ANALYSIS

PLAN	ELEVATION	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM	
				STORAGE	OUTFLOW
1	952.40	0.	0.	0.	0.
RATIO OF RESERVOIR TO W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS
0.3	952.43	0.70	2.	6.	16.92
0.5	953.05	0.05	3.	10.	2.50
0.9	953.36	0.36	5.	16.	5.17
1.0	953.54	0.54	6.	25.	6.92
1.5	954.79	1.79	15.	344.	15.42
1.00	955.49	2.49	21.	753.	16.83

COMPUTER SUMMARY ANALYSIS

DATE
ILME